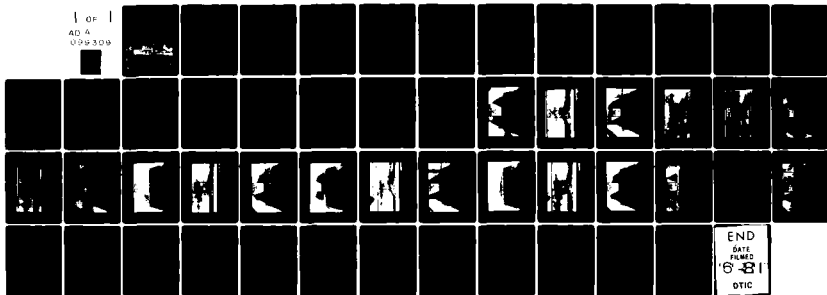
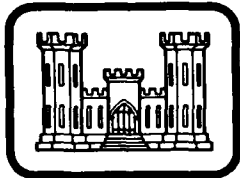


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RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS, P--ETC(U)
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TECHNICAL REPORT HL-81-5

RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION TESTS PORT ONTARIO HARBOR, NEW YORK

Hydraulic Model Investigation

by

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April 1981
Final Report

Approved For Public Release; Distribution Unlimited



Prepared for U. S. Army Engineer District, Buffalo
Buffalo, New York 14207

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20. ABSTRACT (Continued).

with two thicknesses of 7.8-ton stone. Plan 1A was the same as Plan 1 except that the crown elevation was lowered to +9 ft lwd. Plan 2 was similar to Plan 1 except that the armor weight was reduced to 5.3 tons and the crown width was narrowed to 14 ft. Based on results of model tests, it was concluded that Plans 1 and 2 meet the designated wave-transmission criteria of significant transmitted wave height ≤ 3.0 ft and are stable designs for the maximum breaking wave heights that can be produced in the model for 7- to 11-sec waves at swl's of +1.0 and +4.6 ft lwd. Plan 1 exhibited the best stability response of all three plans investigated. Maximum significant transmitted wave heights were 2.5, 3.0 and 2.4 ft for Plans 1, 1A, and 2, respectively.

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PREFACE

The model investigation reported herein was requested by the U. S. Army Engineer District, Buffalo (NCB), in a letter to the U. S. Army Engineer Waterways Experiment Station (WES) dated 2 May 1980. Funding authorization was granted by NCB on Intra-Army Order No. NCB-IA-80-58JD, dated 27 May 1980.

Model tests were conducted at WES during the period July 1980 to September 1980, under the general direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory, and Dr. R. W. Whalin, Chief of the Wave Dynamics Division. Tests were conducted by Messrs. R. D. Carver and D. G. Markle, Research Hydraulic Engineers; Mr. M. S. Taylor, Engineering Technician; and Mrs. B. J. Wright, Engineering Aid. Execution of this study and preparation of this report was performed by Messrs. Carver and Markle under the supervision of Mr. D. D. Davidson, Chief of the Wave Research Branch.

Liaison between NCB and WES was maintained during the course of the investigation by telephone communication and progress reports.

Commander and Director of WES during the conduct of the study and the preparation and publication of this report was COL Nelson P. Conover, CE. Technical Director was Mr. F. R. Brown.

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CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI)
UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

<u>Multiply</u>	<u>By</u>	<u>To Obtain</u>
feet	0.3048	metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
tons (2000 lb mass)	907.1847	kilograms

RUBBLE-MOUND BREAKWATER STABILITY AND WAVE-ATTENUATION

TESTS, PORT ONTARIO HARBOR, NEW YORK

Hydraulic Model Investigation

PART I: INTRODUCTION

Background

1. Port Ontario Harbor, New York, is situated at the mouth of the Salmon River, about 20 miles* south of the entrance to Henderson Bay, on the eastern shore of Lake Ontario (Plate 1). The area tributary to Port Ontario Harbor is principally recreational and agricultural with the village of Port Ontario, 1 mile upstream, catering to summer vacationers. At present, a constantly shifting sand and cobble bar, caused mainly by littoral drift due to wave action, poses numerous navigation problems.

2. A plan of improvement for Port Ontario Harbor based on hydraulic model tests,** fiscal considerations, and local interests has been formulated by the U. S. Army Engineer District, Buffalo (NCB). Proposed improvements include dredging an entrance channel which will be protected by a rubble-mound breakwater. Design constraints dictate that the crest elevation of the breakwater not exceed +10 ft low water datum (lwd); therefore, it is anticipated that major wave overtopping will occur and may cause instability of the breakwater's back slope. Design criteria require that the significant transmitted waves do not exceed 3.0 ft.

Purpose of Model Study

3. The original purpose of the model study was to experimentally

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 3.

** R. R. Bottin, Jr. 1977 (Nov). "Port Ontario Harbor, New York, Design for Wave Protection and Prevention of Shoaling; Hydraulic Model Investigation," Technical Report H-77-20, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

investigate the armor stability and wave transmission properties of a breakwater section proposed by NCB for use at Port Ontario Harbor. Plan 1 used 7.8-ton armor stone, a crown width of 16 ft, and armor slopes of 1V on 2H and 1V on 1.5H lakeside and harbor side, respectively. Later, following completion of tests for Plan 1, it was decided to investigate two alternate plans in an attempt to reduce construction costs for the breakwater.

PART II: THE MODEL

Design of Model

4. Tests were conducted at an undistorted linear scale of 1:28, model to prototype. Scale selection was based on the size of model armor units available compared with the estimated size of prototype armor units required for stability, the elimination of stability scale effects,* and capabilities of the available wave tank. Based on Froude's model law** and the linear scale of 1:28, the following model-prototype relations were derived. Dimensions are in terms of length (L) and time (T).

<u>Characteristics</u>	<u>Dimensions</u>	<u>Model:Prototype Scale Relations</u>
Length	L	$L_r = 1:28$
Area	L^2	$A_r = L_r^2 = 1:784$
Volume	L^3	$V_r = L_r^3 = 1:21,952$
Time	T	$T_r = L_r^{1/2} = 1:5.29$

5. The specific weight of water used in model tests was assumed to be the same as the prototype and equal to 62.4 pcf. However, specific weights of model breakwater construction materials were not the same as their prototype counterparts. These variables were related using the following transference equation:

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left(\frac{L_m}{L_p} \right)^3 \left[\frac{(S_r)_p - 1}{(S_r)_m - 1} \right]^3$$

* R. Y. Hudson. 1975 (Jun). "Reliability of Rubble-Mound Breakwater Stability Models," Miscellaneous Paper H-75-5, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

** J. C. Stevens et al. 1942. "Hydraulic Models," Manuals on Engineering Practice No. 25, American Society of Civil Engineers, New York, N. Y.

where subscripts m and p represent model and prototype quantities, respectively, and

- W_r = weight of an individual armor unit or stone, lb
- γ_r = specific weight of an individual armor unit or stone, pcf
- L_m/L_p = linear scale of the model
- S_r = specific gravity of an individual armor unit or stone relative to the water in which the breakwater is constructed, i.e., $S_r = \gamma_r/\gamma_w$ where γ_w = the specific weight of water, pcf

Test Equipment

6. Tests were conducted in a portion of an L-shaped concrete flume (100 ft long, 5 ft wide, and 3 ft deep) which has overall dimensions of 250 ft long, 50 and 80 ft wide at the top and bottom of the L, respectively, and 4.5 ft deep. The flume layout is shown in Plate 2. A 44-ft (model) length of 1V-on-50H slope, representative of the existing prototype sea bottom, was molded and test sections were installed 20 ft beachward of the slope's toe. The test facility is equipped with a flap-type wave generator, capable of producing sinusoidal waves of various periods and heights. Tests waves of the required characteristics were generated by varying the frequency and amplitude of the plunger motion. Changes in water-surface elevation, as a function of time, were measured by electrical wave-height gages in the vicinity of where the toe of the test section was to be placed and recorded on chart paper by an electrically operated oscillograph. Wave-generator calibration, without the test section in place, simulated existing conditions.

PART III: TESTS AND RESULTS

Method of Constructing Test Sections

7. Model breakwater sections were constructed to reproduce as closely as possible results of the usual methods of constructing prototype structures. Core material, dampened as it was dumped by bucket or shovel into the flume, was compacted with hand trowels to simulate natural consolidation resulting from wave action during construction of the prototype breakwater. Once the core material was in place, it was sprayed with a low-velocity water hose to ensure adequate compaction of the material. Underlayer stone then was added by shovel and smoothed to grade by hand or with trowels but was not packed in place. Armor units used in the cover layer were placed by hand in a random manner (i.e., laid down in such a way that no intentional interlocking of the units was obtained). Model elevations were controlled with an engineer's level to a tolerance of ± 0.005 ft.

Description of Plan 1

8. Plan 1 (Plate 3 and Photos 1-3) was constructed to a crown elevation of +10 ft lwd and used armor slopes of 1V on 2H and 1V on 1.5H lakeside and harbor side, respectively. A crown width of 16 ft, equivalent to three armor-stone diameters, was used; and the slopes and crown were armored with two thicknesses of 7.8-ton stone. A 10-ft-wide berm of 1560-lb underlayer stone was placed at the toe of both the lakeside and harbor-side slopes.

Selection of Test Conditions

9. Based on anticipated prototype wave conditions,* it was

* D. T. Resio and C. L. Vincent. 1976 (Mar). "Design Wave Information for the Great Lakes; Lake Ontario," Technical Report H-76-1, Report 2, U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

decided that the stability tests should consider 7-, 9-, and 11-sec waves at still water levels (swl's) of +1.0 and +4.6 ft lwd. Model observations indicated that for the selected wave periods and swl's, the corresponding maximum breaking wave was always more severe than any lesser wave height. Observations of incident wave forms at the structure showed the most severe breaking wave conditions that experimentally could be made to attack the section for the selected conditions were as follows:

<u>Swl</u> <u>ft lwd</u>	<u>Wave Period</u> <u>sec</u>	<u>Most Severe Breaking</u> <u>Wave Height, ft</u>
+1.0	7.0	6.1
+1.0	9.0	6.8
+1.0	11.0	7.0
+4.6	7.0	7.9
+4.6	9.0	9.3
+4.6	11.0	9.8

It was decided that for the range of wave conditions considered, the stability response of the proposed test sections could be adequately evaluated by subjecting the structure to the following storm-surge hydrograph:

<u>Step</u>	<u>Swl</u> <u>ft lwd</u>	<u>Test Wave</u> <u>(Prototype Dimensions)</u>		<u>Prototype</u> <u>Duration, hr</u>	<u>Wave Type</u>
		<u>Period, sec</u>	<u>Height, ft</u>		
	+1.0	7.0	3.0	0.33	Shakedown
1	+1.0	7.0	6.1	0.50	Severe breaking
2	+1.0	9.0	6.8	0.50	Severe breaking
3	+1.0	11.0	7.0	1.00	Severe breaking
4	+4.6	7.0	7.9	0.50	Severe breaking
5	+4.6	9.0	9.3	0.50	Severe breaking
6	+4.6	11.0	9.8	1.00	Severe breaking
7	+1.0	7.0	6.1	0.50	Severe breaking
8	+1.0	9.0	6.8	0.50	Severe breaking
9	+1.0	11.0	7.0	1.00	Severe breaking

10. Wave-attenuating capabilities of the proposed breakwater sections also were investigated. In these tests, transmitted wave heights were measured at distances of one-half wavelength ($L/2$) and one wavelength (L) behind the center line of the breakwater for 7-, 9-, and 11-sec waves at an swl of +4.6 ft lwd. Incident wave heights ranged from 5 ft up to the maximum depth-limited waves that could reach the

structure for the selected wave periods and swl.

Test Results of Plan 1

11. Plan 1 exhibited an excellent stability response. Moderate wave overtopping, present during steps 4, 5, and 6 of the test hydrograph, caused minor rocking of a few 7.8-ton, harbor-side armor units; however, no displacement damage occurred. The 1560-lb, toe-protection stone proved to be stable. Photo 4 shows an 11-sec, 9.8-ft wave (step 6) impinging on the breakwater; and Photo 5 shows the wave overtopping the structure. Photos 6-8 show the structure after exposure to the hydrograph. Stability test results of Plan 1 were verified by a complete reconstruction and retesting.

12. Wave-attenuation test results are presented in Table 1 and Plates 4 and 5. The values presented therein are the measured significant wave heights. Detailed analysis of the wave records showed that for a given test condition, the maximum transmitted wave height never exceeded the significant transmitted wave height by more than 10 percent. In general, the data show that if the incident wave height were held constant and the wave period were increased, transmitted heights generally increased; and transmitted wave heights measured at $L/2$ were slightly larger than those observed at a distance L behind the breakwater. High-frequency disturbances (which quickly dissipated), produced by incident waves overtopping and transmitting through the porous armor stone, probably accounted for the slightly larger transmitted heights observed one-half wavelength behind the breakwater. Typical transmitted wave forms are illustrated in Plate 6.

Rationale and Description of Plans 1A and 2

13. Based on the excellent stability response of Plan 1, it was decided to investigate alternative schemes that might substantially reduce the structure's cost without significantly affecting its functional performance. Some of the factors that govern material volumes and costs

are elevation and width of the crown, type and weight of armor, and slope on which the armor is placed. Based on discussions between NCB and U. S. Army Engineer Waterways Experiment Station, it was decided that in this particular study the greatest cost savings with the least probable impact on functionality could probably be achieved by either lowering the crown elevation or reducing the armor weight. Thus, two additional plans (Plans 1A and 2) were selected for testing. Plan 1A (Plate 3 and Photos 9-11) was the same as Plan 1 except that the crown elevation was lowered to +9 ft lwd. Plan 2 (Plate 7 and Photos 12-14) was similar to Plan 1 except that the armor weight was reduced to 5.3 tons and the crown width was narrowed to 14 ft. Also, the underlayer and toe-protection stone weight was proportionately reduced to 1060 lb.

Test Results of Plans 1A and 2

14. Plan 1A demonstrated a good stability response. Relative to Plan 1, increased wave overtopping was observed during steps 4, 5, and 6 of the test hydrograph. Attack of 11-sec, 9.8-ft waves (step 6) displaced one harbor-side armor unit downslope; however, this displacement had no effect on the overall stability of the breakwater. Photos 15-17 show the after-testing condition of the structure.

15. Plan 2 also demonstrated a good stability response. No armor movement was detected for any of the low-water conditions (hydrograph steps 1, 2, 3, 7, 8, and 9); however, rocking of a few 5.3-ton armor units was observed during hydrograph steps 4, 5, and 6. Attack of 11-sec, 9.8-ft waves (step 6) displaced two lakeside armor units onto the 1060-lb toe-protection stone. This displacement had no impact on the overall integrity of the breakwater, and the final stabilized condition was deemed to be completely acceptable. Photos 18-20 show the breakwater after completion of the hydrograph.

16. The 1560-lb and 1060-lb toe-protection stone used in Plans 1A and 2, respectively, proved to be stable. Stability test results of both Plans 1A and 2 were verified by complete reconstructions and retestings.

17. Wave-attenuation test results (significant transmitted wave

heights) are presented in Tables 2 and 3 and Plates 8-11. These data show the same general trends as those observed with Plan 1. If the incident wave height were held constant and the wave period increased, transmitted heights increased; and transmitted wave heights measured at $L/2$ were slightly larger than those observed at a distance L behind the breakwater. For a given test condition, the maximum transmitted wave height never exceeded the significant transmitted wave height by more than 10 percent. Relative to Plan 1, Plan 1A showed slightly increased wave transmission and Plan 2 showed slightly decreased wave transmission. These results seem very reasonable since Plan 1A was the same as Plan 1 except that the crown elevation was lowered 1 ft, and Plan 2 was similar to Plan 1 except that the reduced armor weight and slightly higher core tended to decrease the permeability of the breakwater.

PART IV: CONCLUSIONS

18. Based on assumptions, tests, and results reported herein, it is concluded that:

- a. Plans 1, 1A, and 2 are stable designs for the maximum breaking wave heights that can be expected to occur for 7- to 11-sec waves at swl's of +1.0 and +4.6 ft lwd.
- b. Plan 1 exhibited the best stability response of all plans investigated.
- c. Maximum significant transmitted wave heights were 2.5, 3.0, and 2.4 ft for Plans 1, 1A, and 2, respectively.

Table 1
Values of Incident and Significant Transmitted Wave Heights
for Plan 1; swl = +4.6 ft lwd

Significant Transmitted			
Wave Height, H_t , ft			
Incident Wave Height,* H_1 , ft	Measured at**		Incident Wave Form
	<u>L/2</u>	<u>L</u>	
<u>7-sec Wave Period</u>			
5.0	1.3	1.2	Nonbreaking
6.0	1.6	1.4	Nonbreaking
7.0	1.7	1.5	Nonbreaking
7.9	1.9	1.7	Breaking
<u>9-sec Wave Period</u>			
5.0	1.3	1.2	Nonbreaking
6.0	1.8	1.4	Nonbreaking
7.0	1.9	1.6	Nonbreaking
8.0	2.1	1.9	Nonbreaking
9.0	2.3	2.0	Nonbreaking
9.3	2.3	2.0	Breaking
<u>11-sec Wave Period</u>			
5.0	1.5	1.4	Nonbreaking
6.0	1.8	1.7	Nonbreaking
7.0	2.0	1.9	Nonbreaking
8.0	2.2	2.1	Nonbreaking
9.0	2.4	2.2	Nonbreaking
9.8	2.5	2.3	Breaking

* Measured at toe of structure without structure in place.

** Measured distance in wavelengths from center line of structure.

Table 2
Values of Incident and Significant Transmitted Wave Heights
for Plan 1A; swl = +4.6 ft lwd

Incident Wave Height,* H_i , ft	Significant Transmitted Wave Height, H_t , ft		Incident Wave Form
	Measured at**		
	<u>L/2</u>	<u>L</u>	
	<u>7-sec Wave Period</u>		
5.0	1.6	1.5	Nonbreaking
6.0	1.9	1.7	Nonbreaking
7.0	2.1	2.0	Nonbreaking
7.9	2.3	2.2	Breaking
	<u>9-sec Wave Period</u>		
5.0	1.7	1.6	Nonbreaking
6.0	2.0	1.8	Nonbreaking
7.0	2.2	2.0	Nonbreaking
8.0	2.3	2.2	Nonbreaking
9.0	2.5	2.3	Nonbreaking
9.3	2.6	2.3	Breaking
	<u>11-sec Wave Period</u>		
5.0	1.8	1.8	Nonbreaking
6.0	2.1	2.0	Nonbreaking
7.0	2.3	2.1	Nonbreaking
8.0	2.6	2.3	Nonbreaking
9.0	2.8	2.5	Nonbreaking
9.8	3.0	2.8	Breaking

* Measured at toe of structure without structure in place.

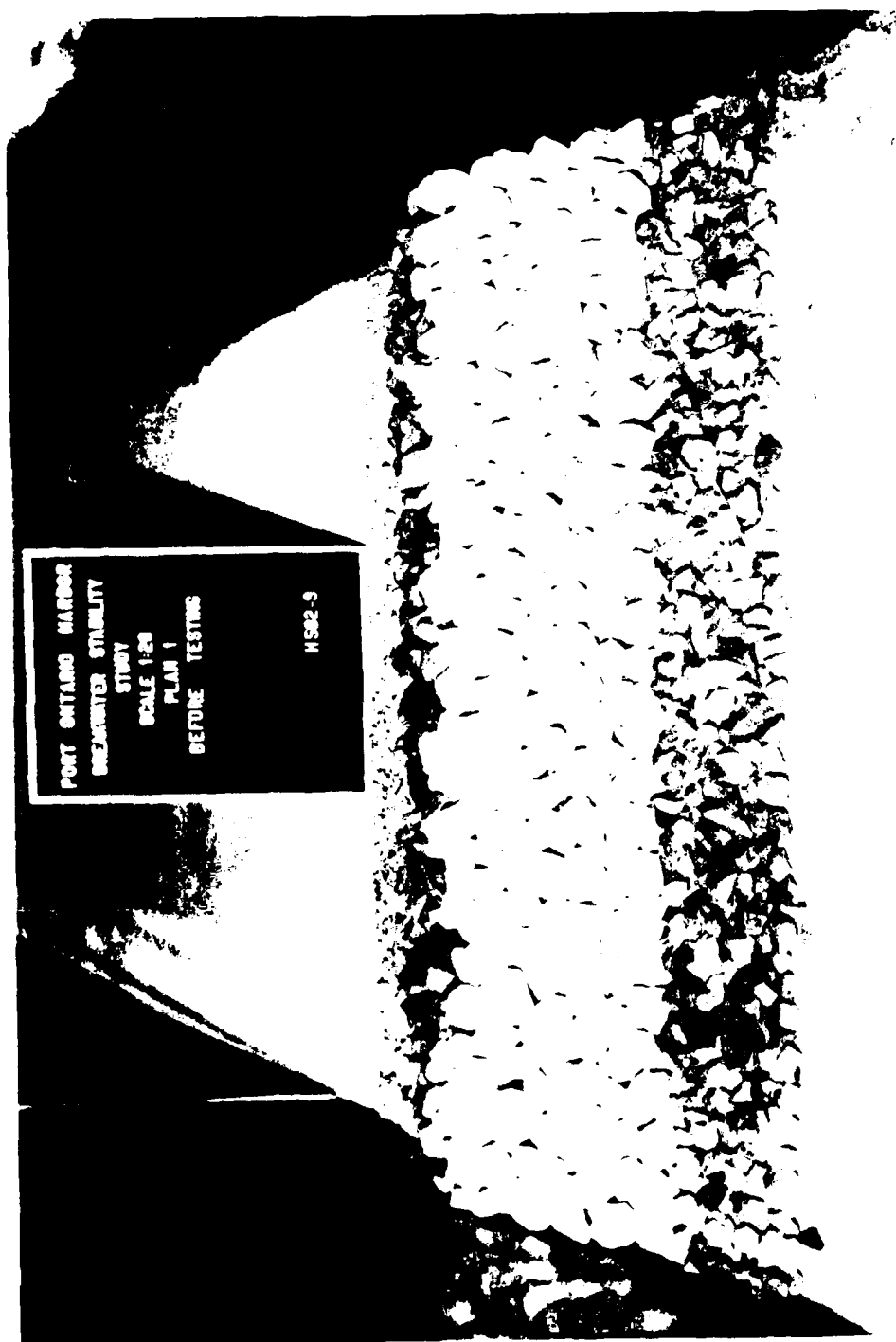
** Measured distance in wavelengths from center line of structure.

Table 3
Values of Incident and Significant Transmitted Wave Heights
for Plan 2; swl = +4.6 ft lwd

Incident Wave Height,* H_i , ft	Significant Transmitted Wave Height, H_t , ft		Incident Wave Form
	Measured at**		
	<u>L/2</u>	<u>L</u>	
<u>7-sec Wave Period</u>			
5.0	1.1	1.0	Nonbreaking
6.0	1.4	1.2	Nonbreaking
7.0	1.5	1.4	Nonbreaking
7.9	1.6	1.5	Breaking
<u>9-sec Wave Period</u>			
5.0	1.2	1.1	Nonbreaking
6.0	1.5	1.3	Nonbreaking
7.0	1.8	1.5	Nonbreaking
8.0	2.0	1.7	Nonbreaking
9.0	2.1	2.0	Nonbreaking
9.3	2.2	2.0	Breaking
<u>11-sec Wave Period</u>			
5.0	1.4	1.4	Nonbreaking
6.0	1.7	1.7	Nonbreaking
7.0	1.9	1.9	Nonbreaking
8.0	2.1	2.0	Nonbreaking
9.0	2.2	2.2	Nonbreaking
9.8	2.4	2.4	Breaking

* Measured at toe of structure without structure in place.

** Measured distance in wavelengths from center line of structure.



PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:20
PLAN 1
BEFORE TESTING

H 582-9

PORT ONTARIO HARBOR BREAKWATER STABILITY STUDY

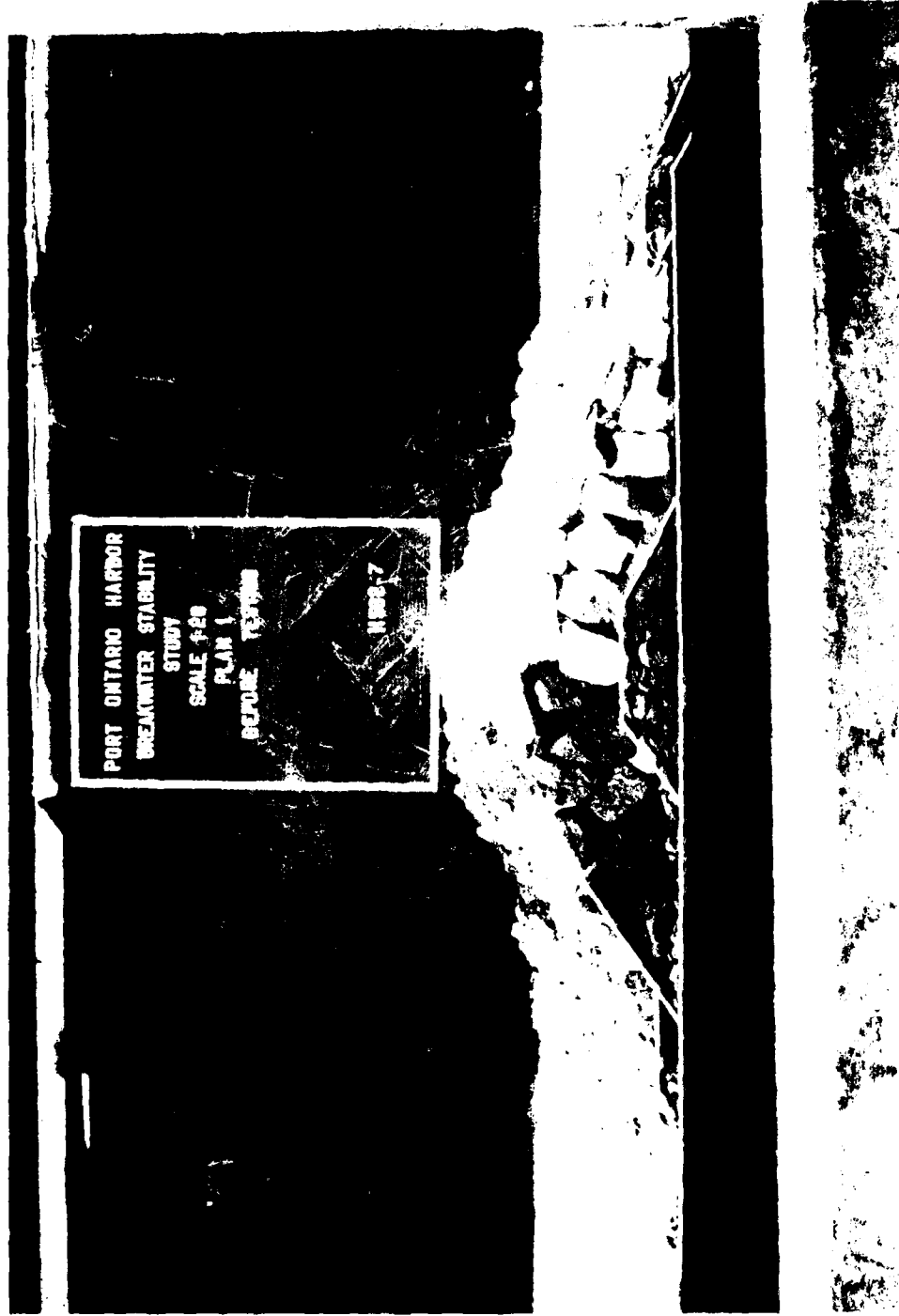


Photo 2. Side view of Plan 1 before wave attack

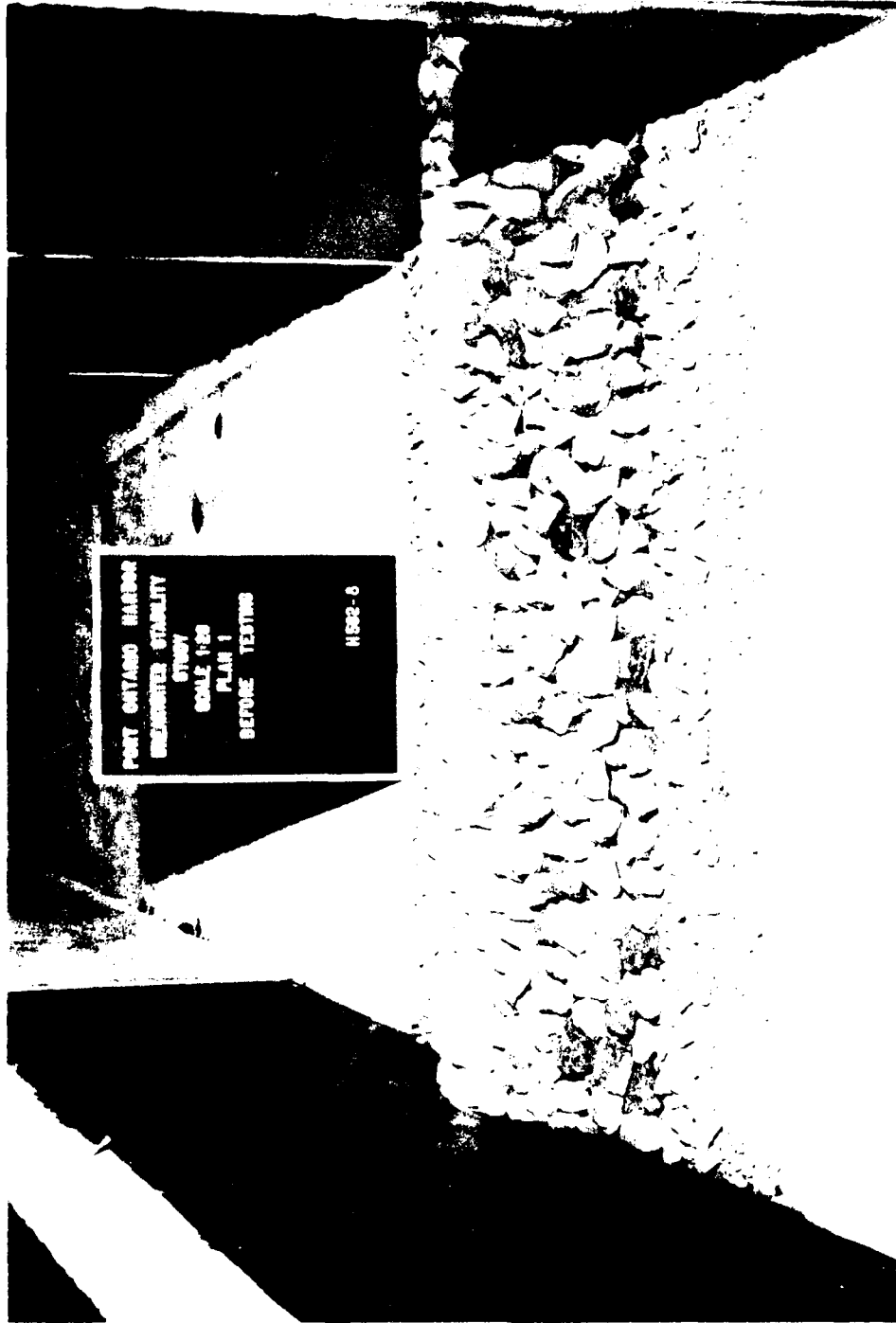


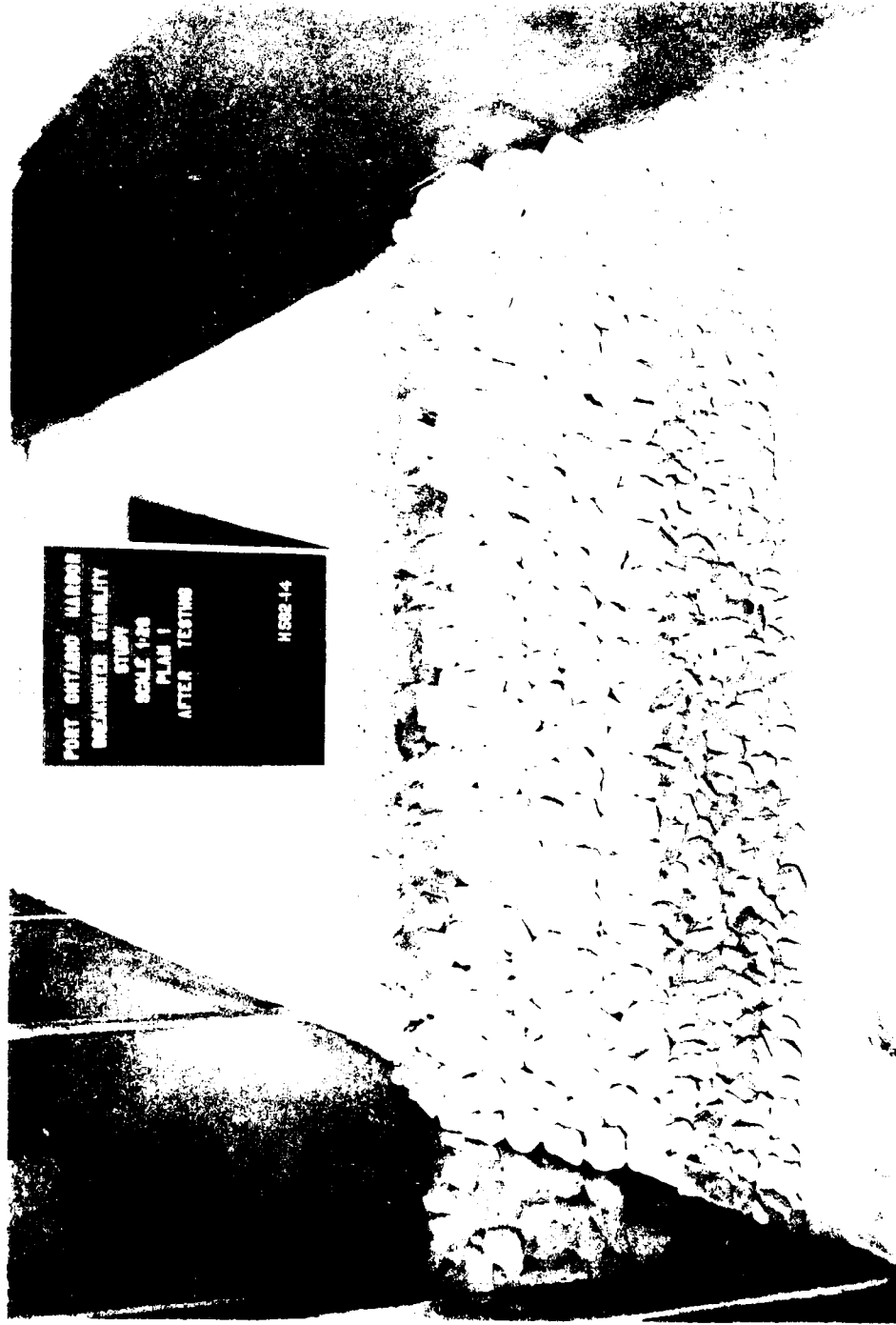
Photo 3. Harbor-side view of Plan 1 before wave attack



Photo 4. Side view of an 11-sec, 9.8-ft wave impinging on Plan I



Photo 5. Side view of Plan 1 being overtopped by an 11-sec, 9.8-ft wave



PORT OTTAWA HARBOUR
BREAKWATER STABILITY
STUDY
SCALE 1/20
PLAN 1
AFTER TESTING

H302-14

Photo 6. Lakeside view of Plan 1 after completion of storm-surge hydrograph



Photo 7. Side view of slip prior completion of storm-surge hydrograph

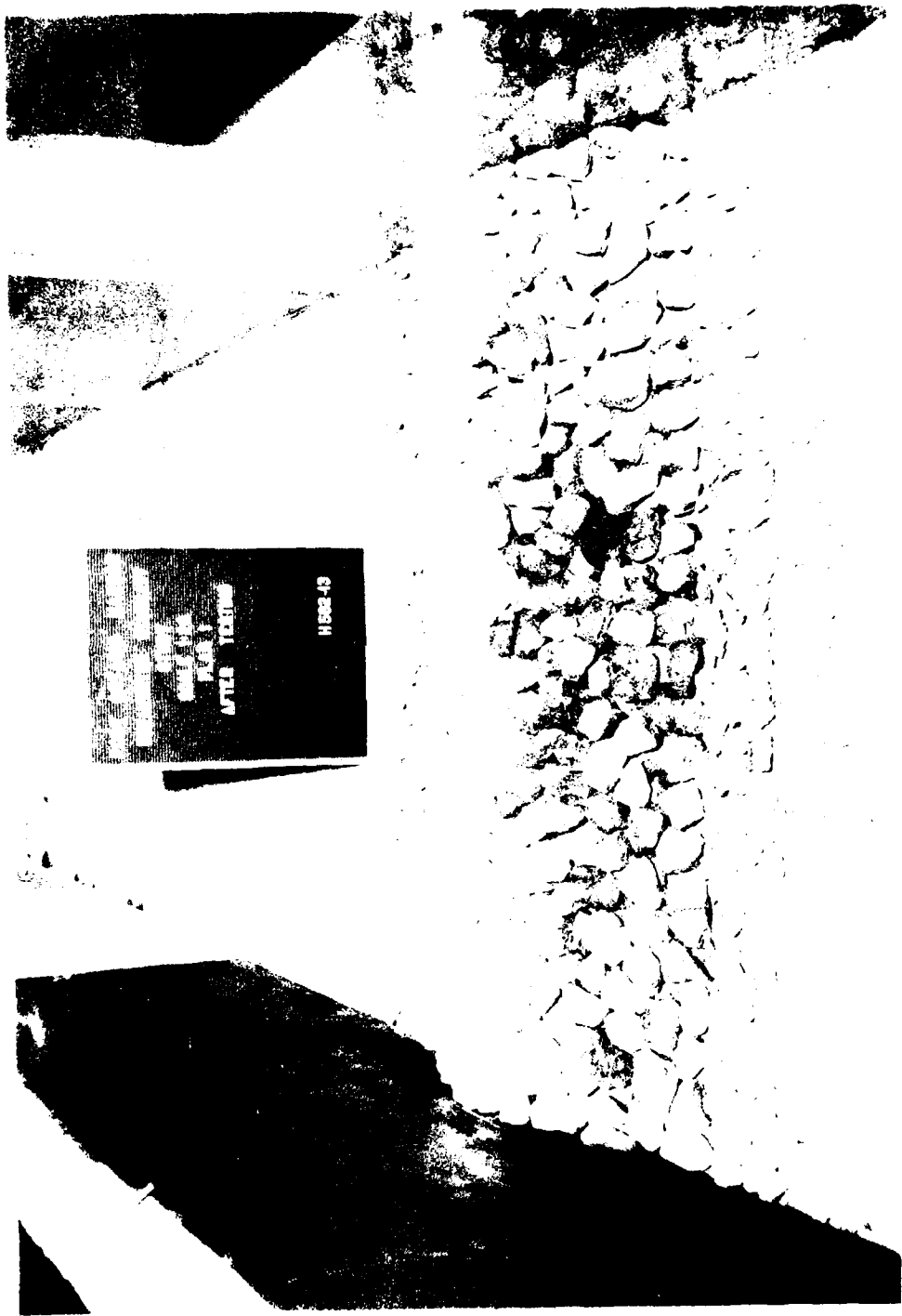


Photo 8. Harbor-side view of Plan 1 after completion of storm-surge hydrograph

PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY

SCALE 1:28

PLAN 1A

BEFORE TESTING

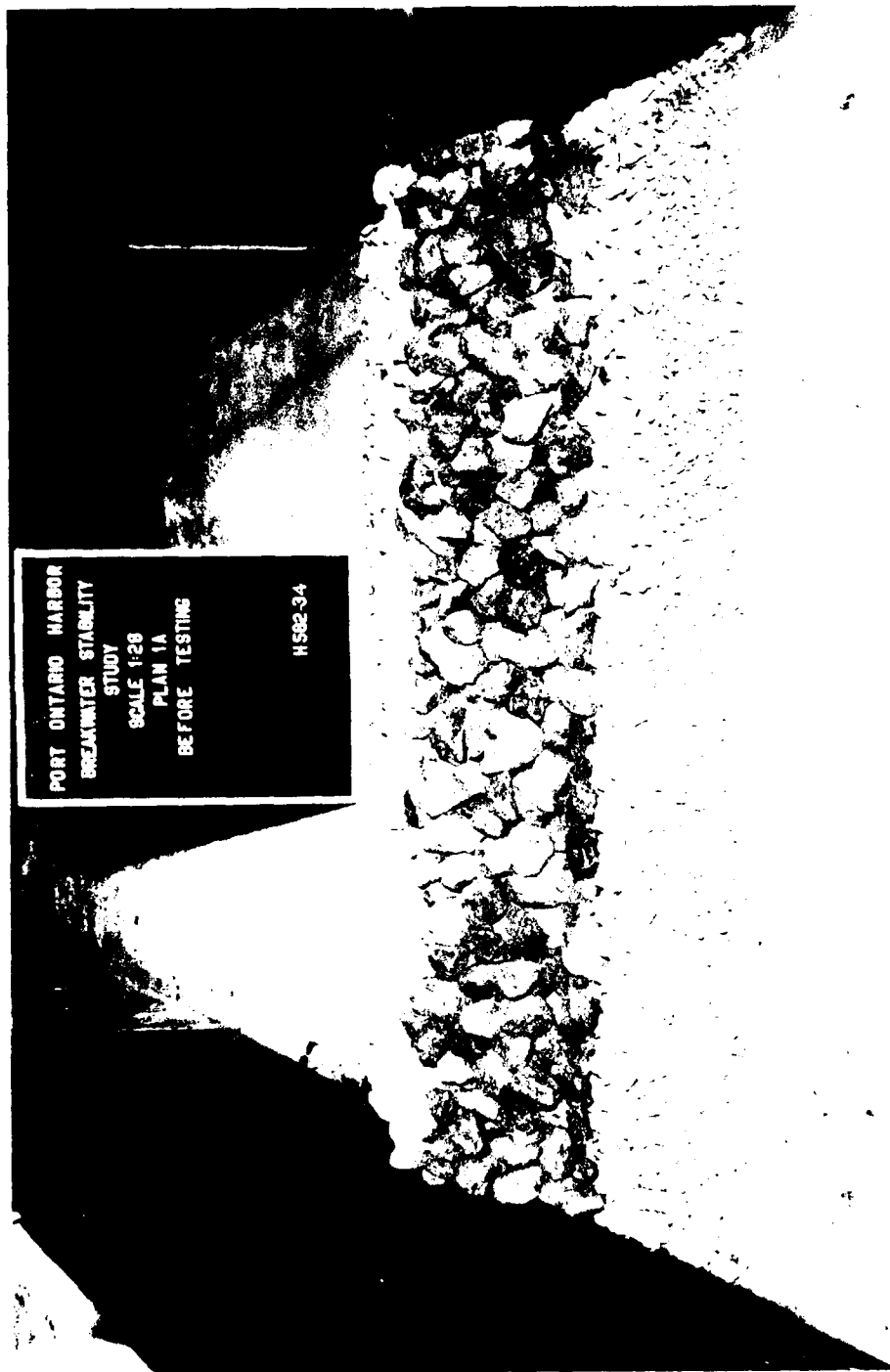
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Photo 9. Aerial view of Plan 1A before wave attack



Photo 10. Side view of Plin 1A before wave attack



PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:28
PLAN 1A
BEFORE TESTING

H582-34

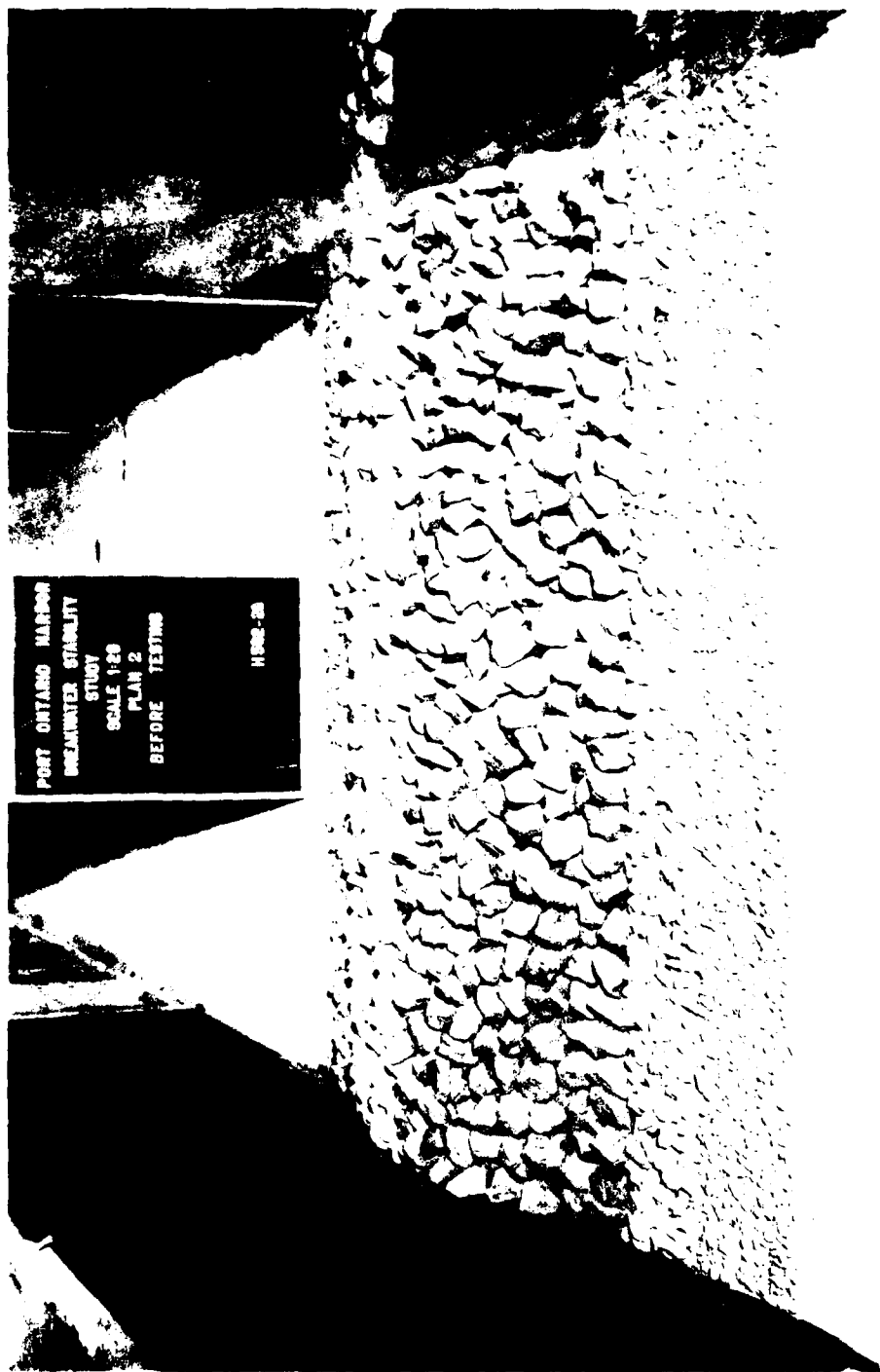
Photo 11. Harbor-side view of Plan 1A before wave attack



Photo 12. Lakeside view of Plan 2 before wave attack



Photo 19. Side view of Plan 1 before wire attack



PORT OTTAWA HARBOR
BREAKWATER STABILITY

STUDY

SCALE 1:25

PLAN 2

BEFORE TESTING

H 1982-20

Photo 14. Harbor-side view of Plan 2 before wave attack.

PORT ONTARIO HARBOR
BREAKWATER STABILITY
STUDY
SCALE 1:20
PLAN 1A
AFTER TESTING

H582 38





Figure 1. Hill Caving, a steep-slope hydrograph

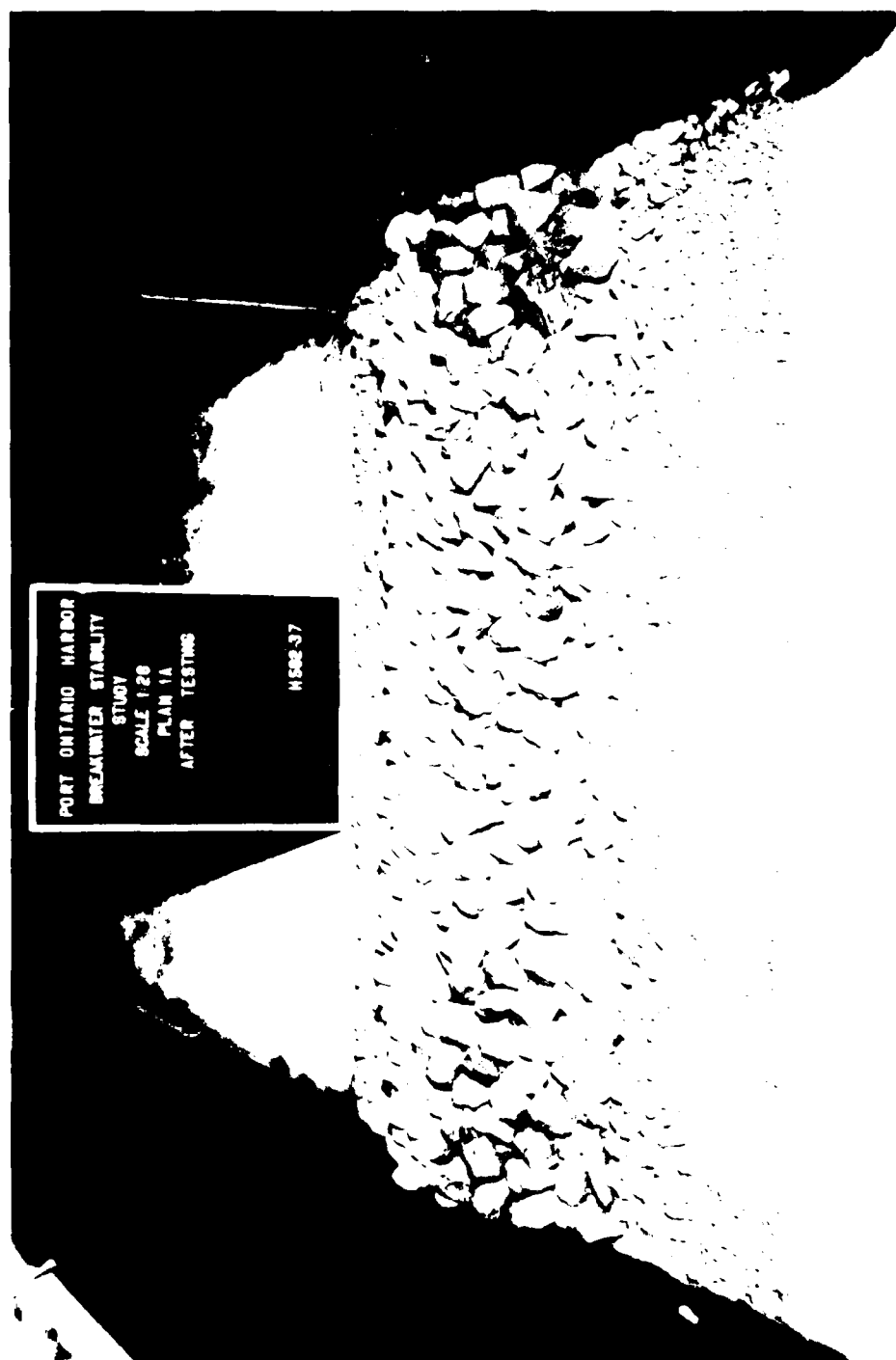
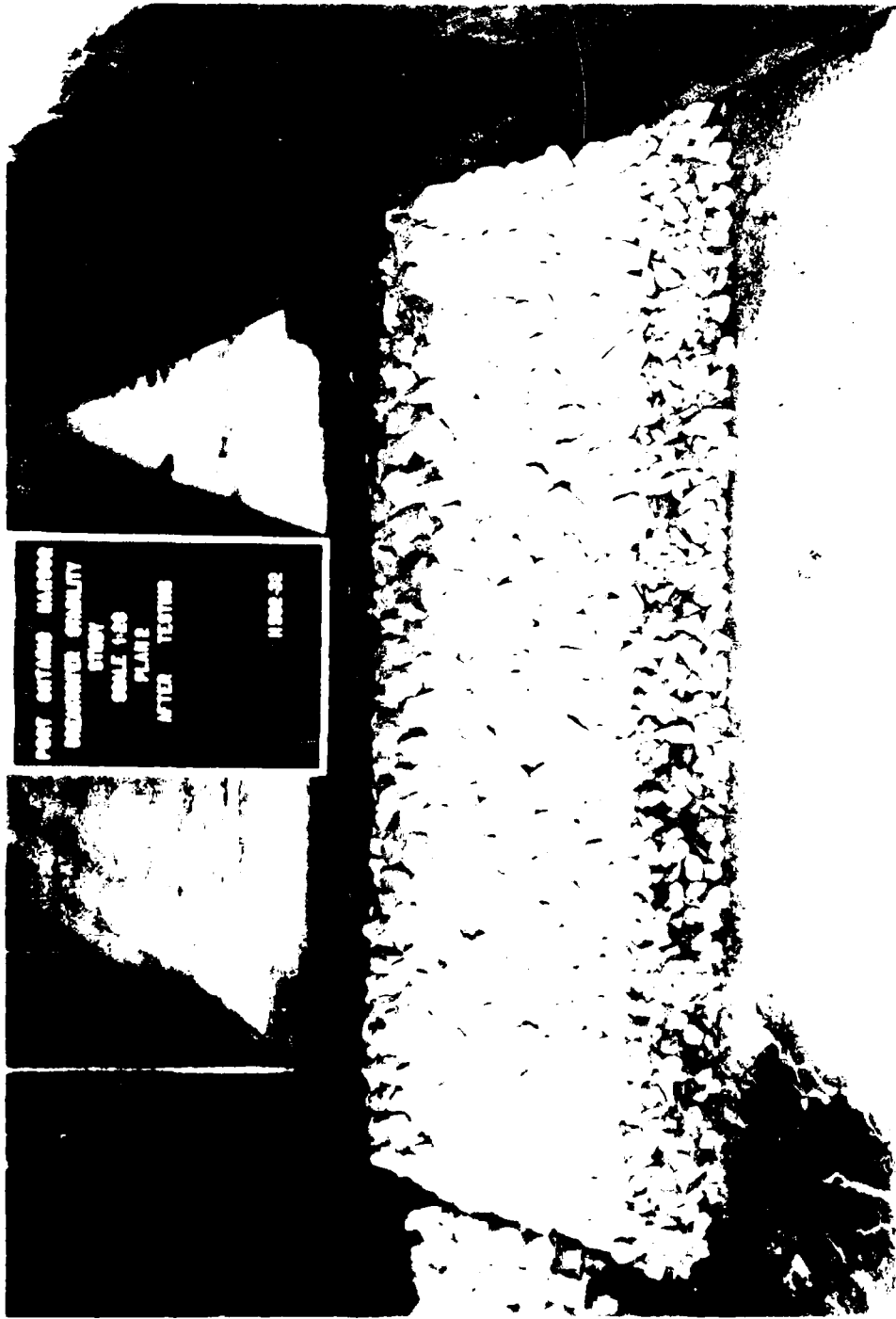


Figure 1. Model breakwater structure and plan view of storm-surge hydrograph

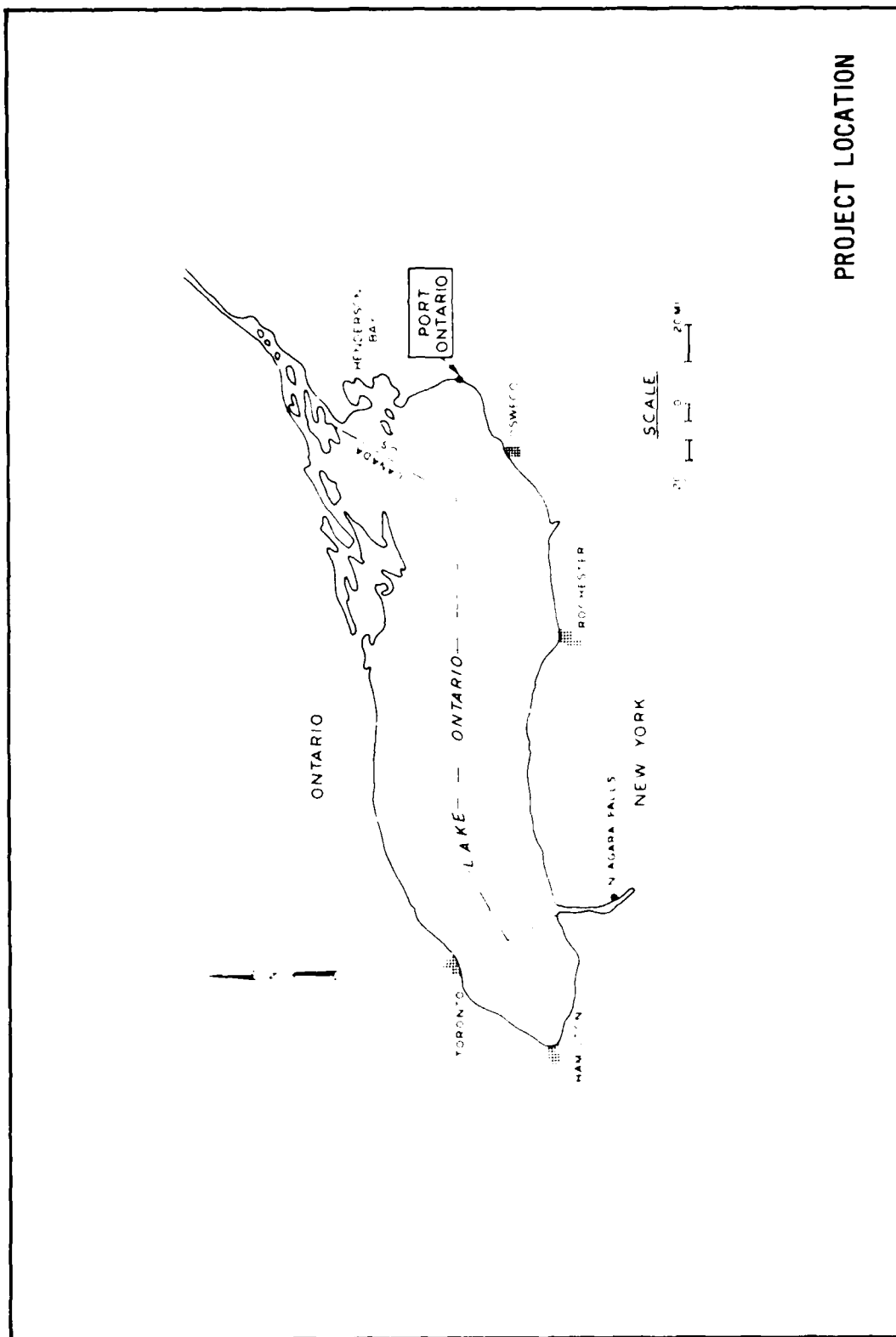


PORT OUTING MARCO
RESEARCHER QUALITY
STUDY
SCALE 1-100
PLATE
AFTER TESTING
1150-22

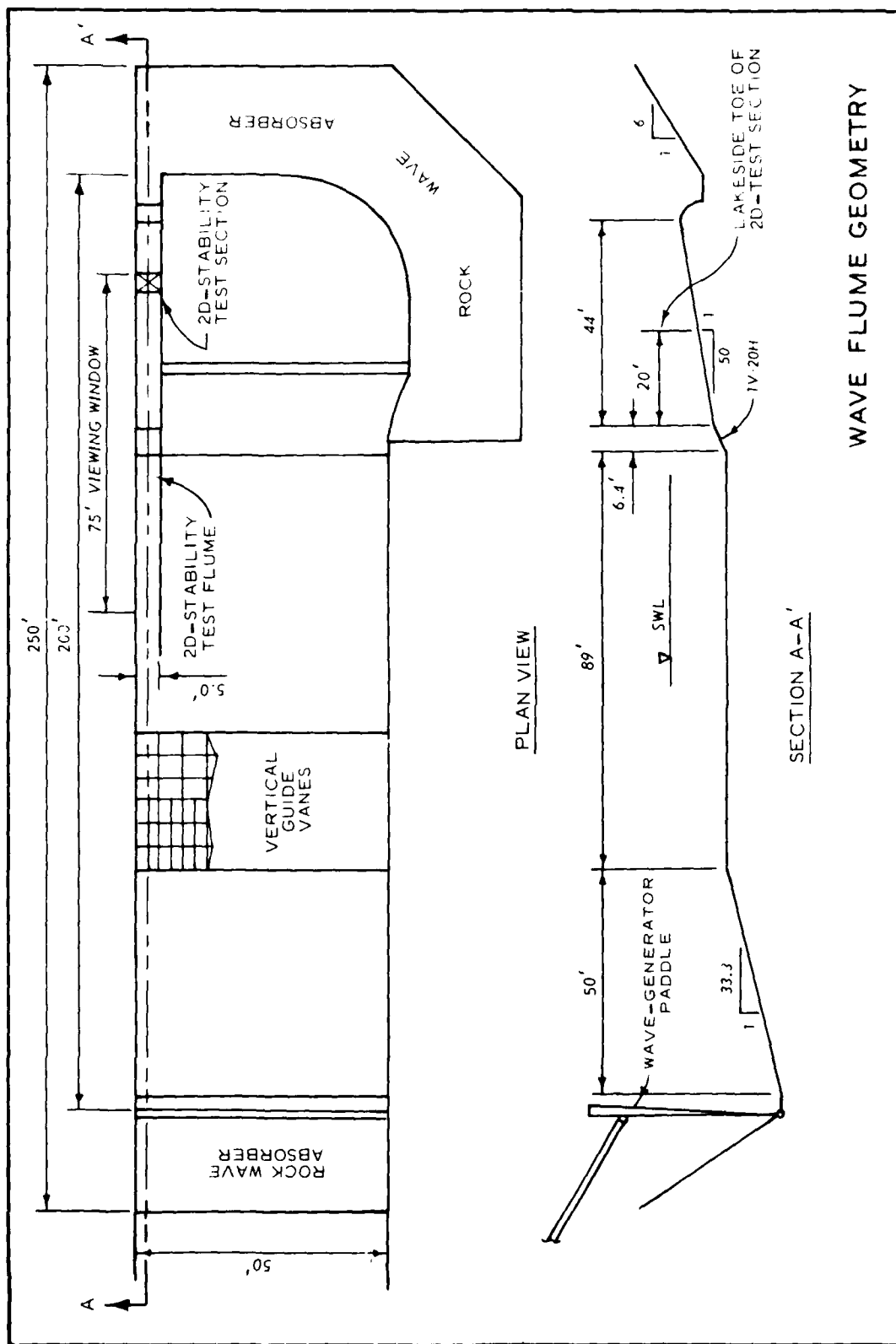




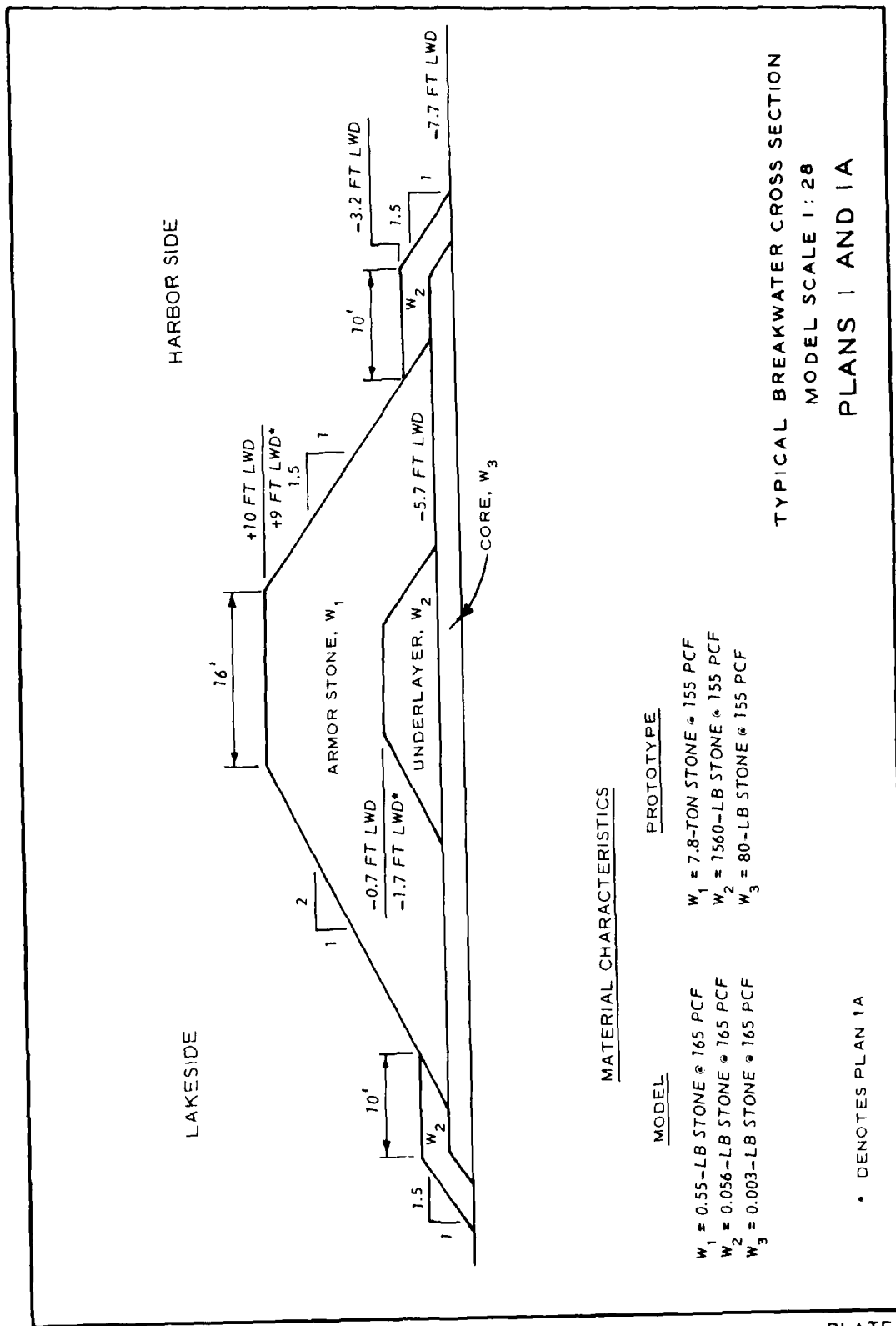
Photo 29. Harbor-side view of Plan 2 after completion of storm-surge hydrograph



PROJECT LOCATION



WAVE FLUME GEOMETRY

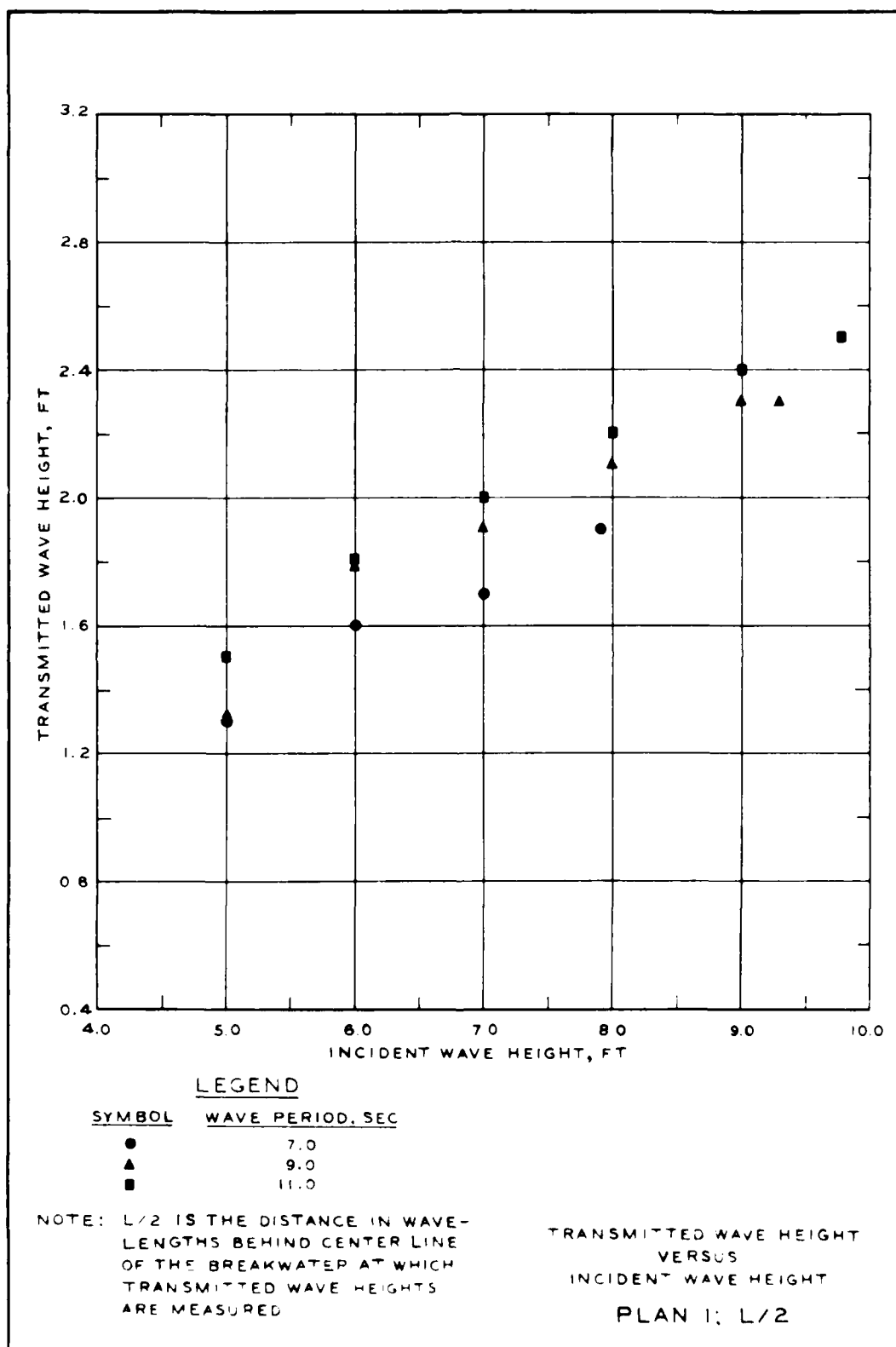


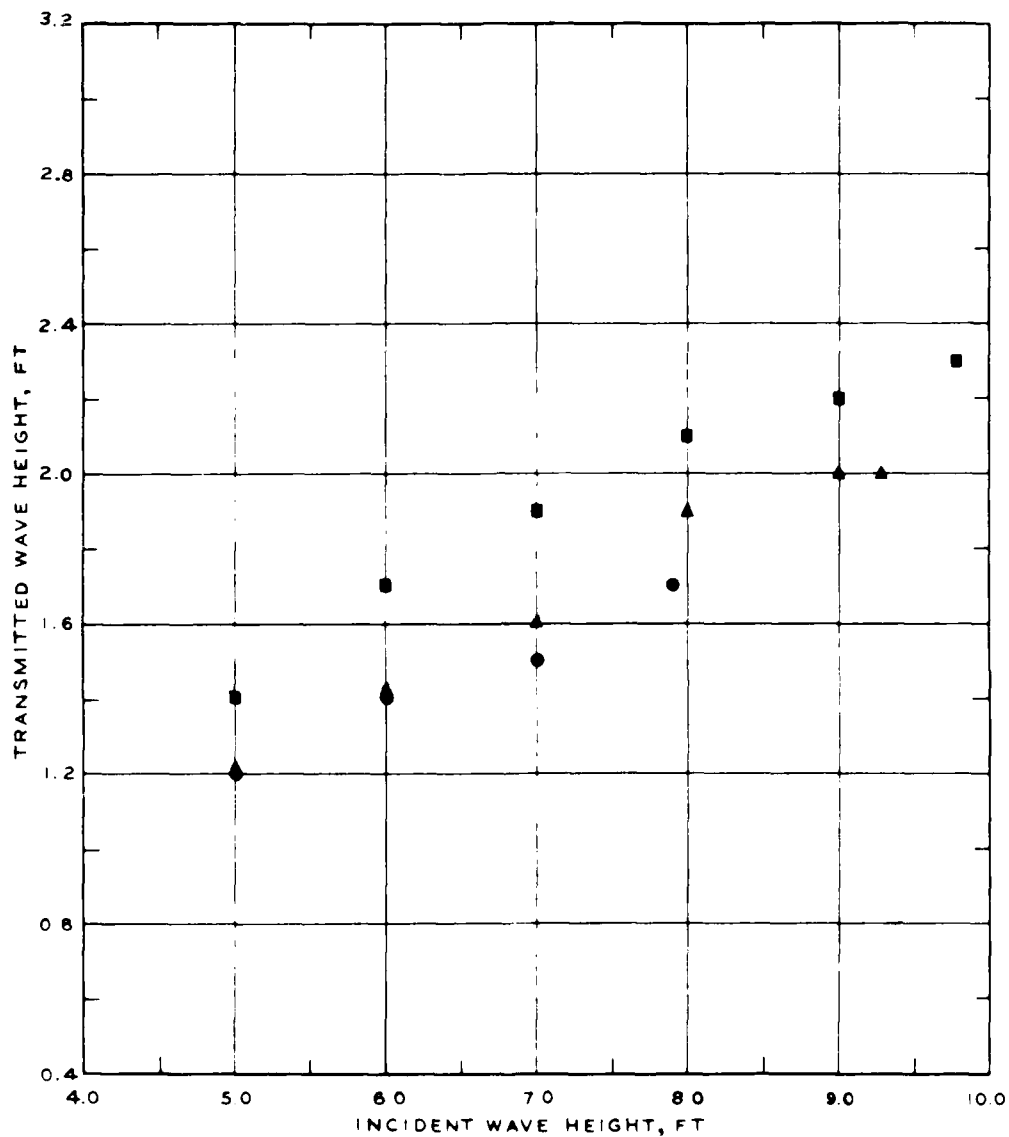
MATERIAL CHARACTERISTICS

MODEL	PROTOTYPE
$W_1 \approx 0.55\text{-LB STONE @ } 165 \text{ PCF}$	$W_1 \approx 7.8\text{-TON STONE @ } 155 \text{ PCF}$
$W_2 \approx 0.056\text{-LB STONE @ } 165 \text{ PCF}$	$W_2 \approx 1560\text{-LB STONE @ } 155 \text{ PCF}$
$W_3 \approx 0.003\text{-LB STONE @ } 165 \text{ PCF}$	$W_3 \approx 80\text{-LB STONE @ } 155 \text{ PCF}$

* DENOTES PLAN 1A

TYPICAL BREAKWATER CROSS SECTION
 MODEL SCALE 1:28
 PLANS 1 AND 1A



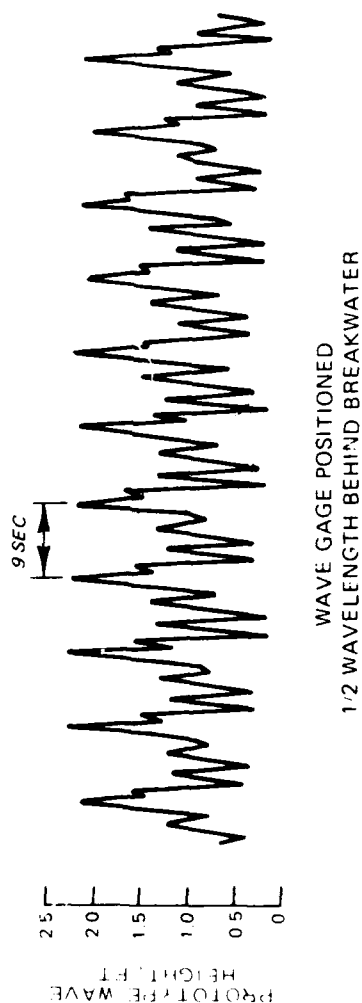


LEGEND

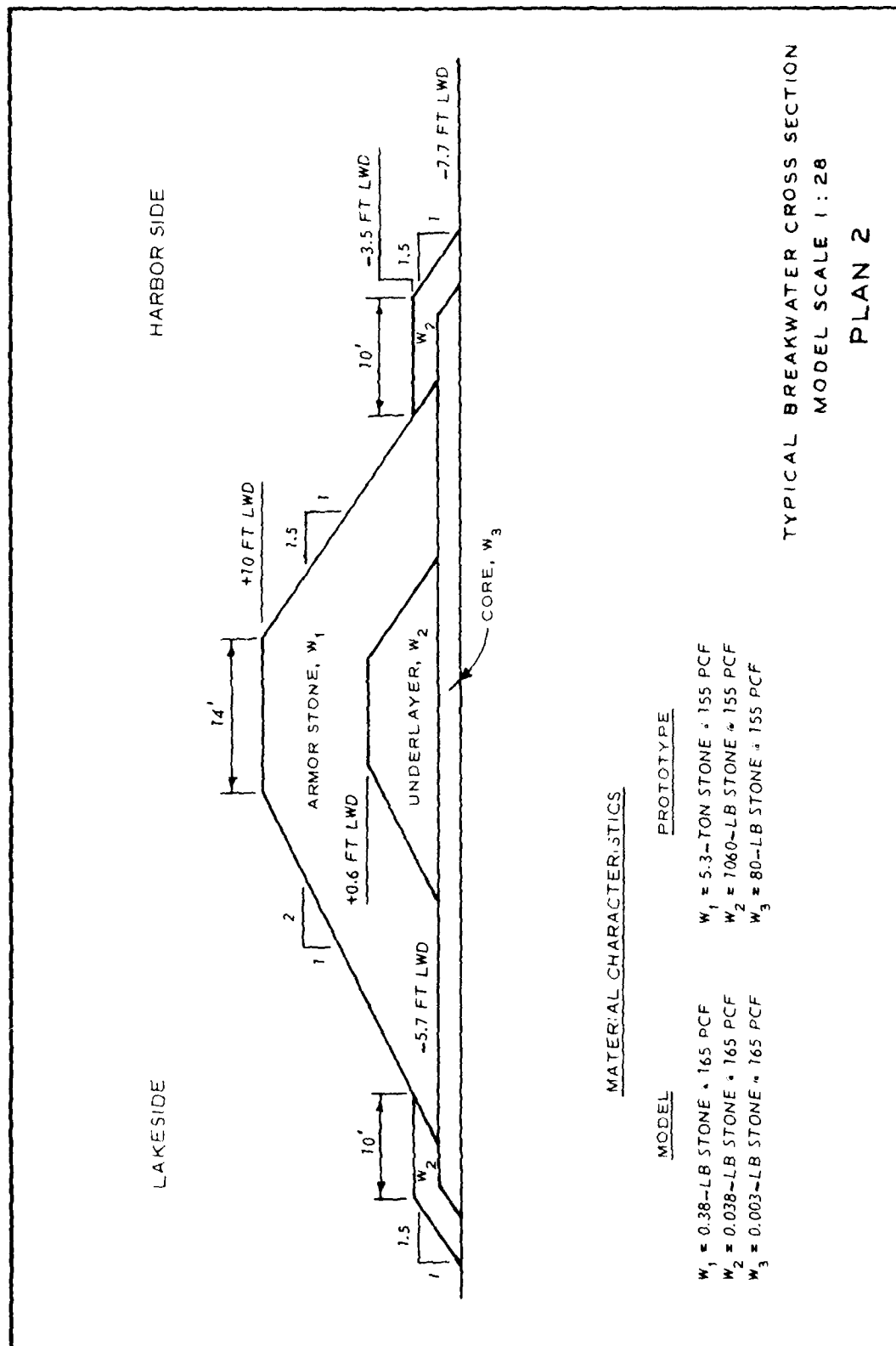
SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

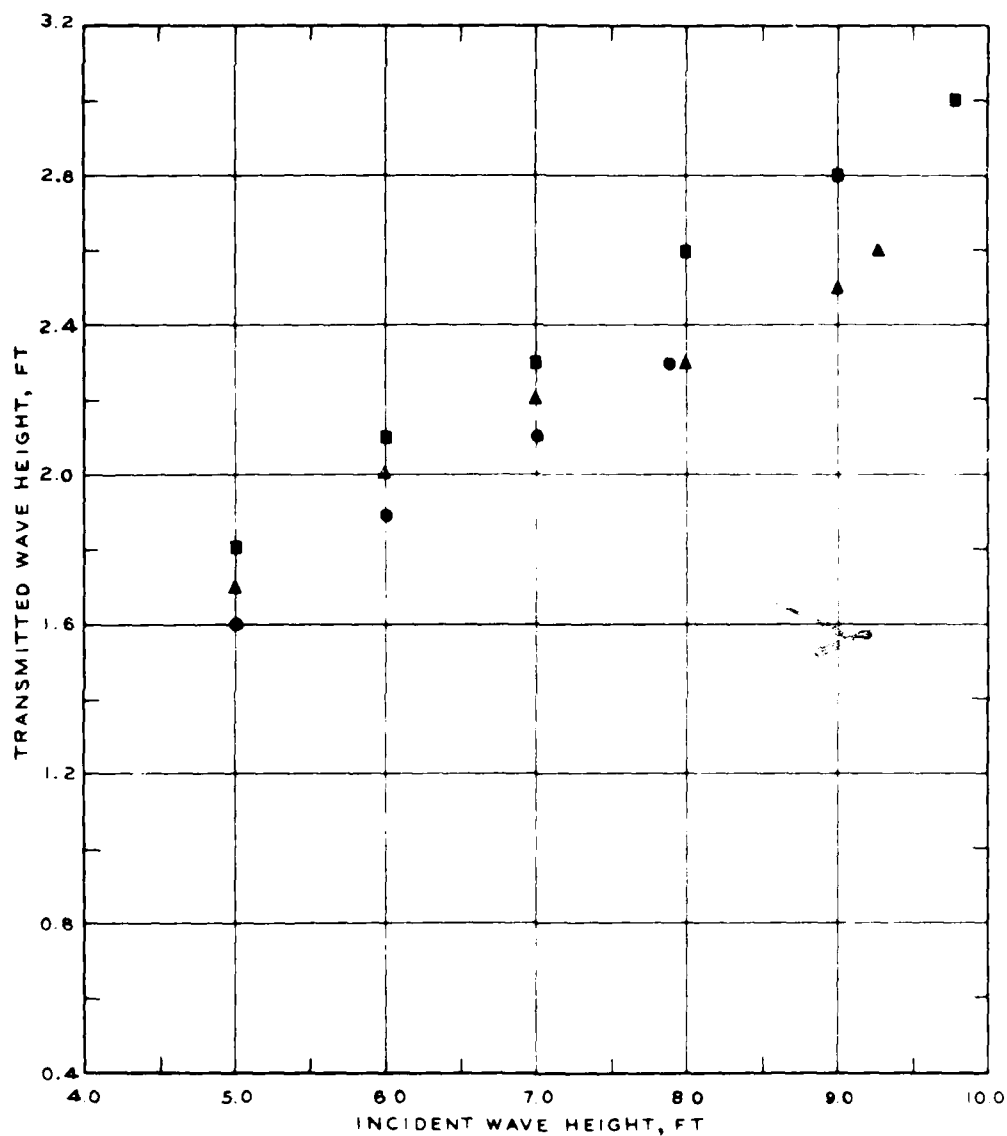
NOTE: L IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT
PLAN I; L



PLAN 1
TYPICAL TRANSMITTED WAVE RECORD
T = 9 SEC; $H_i = 7$ FT



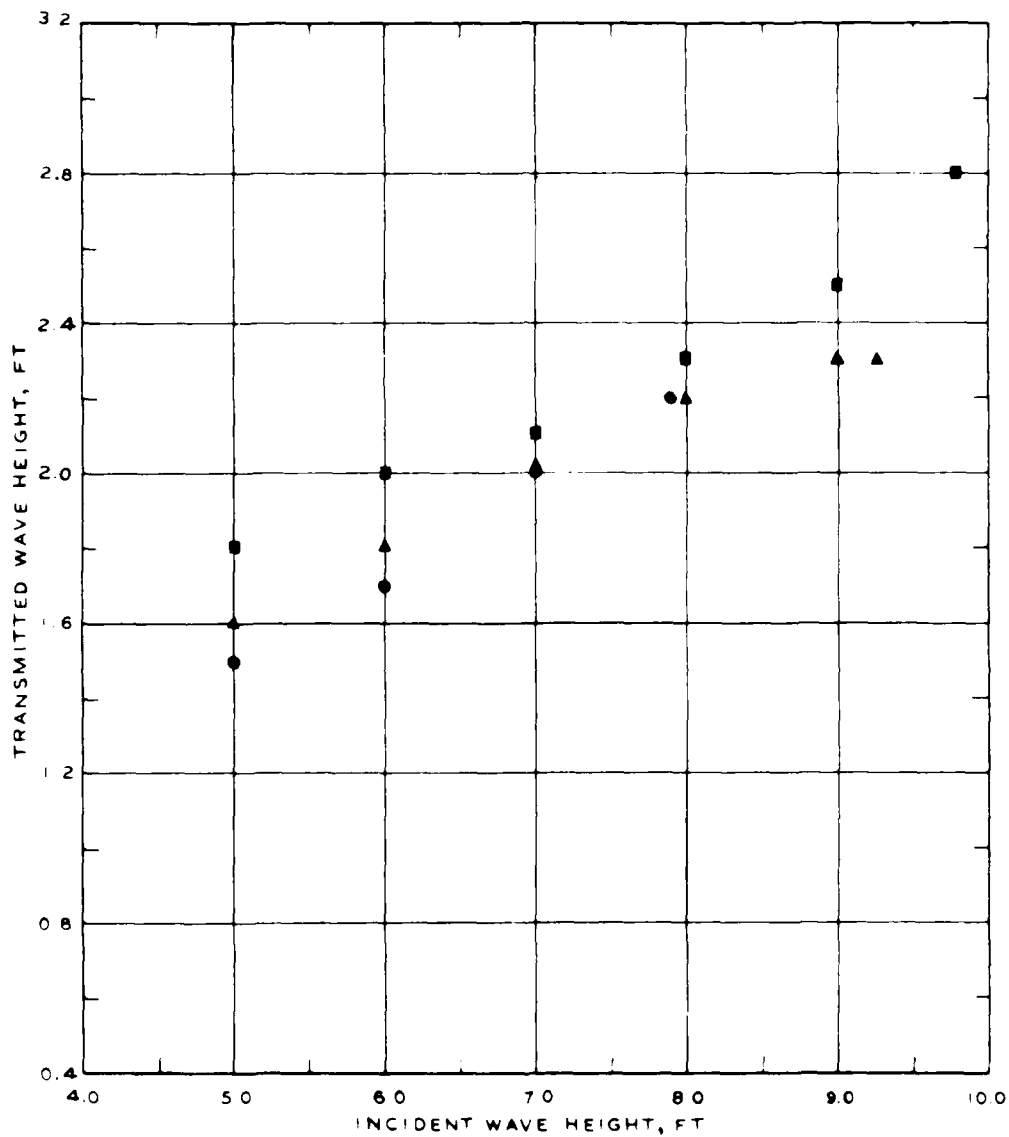


LEGEND

SYMBOL	WAVE PERIOD, SEC
●	7.0
▲	9.0
■	11.0

NOTE: $L/2$ IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED.

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT
PLAN 1A; $L/2$



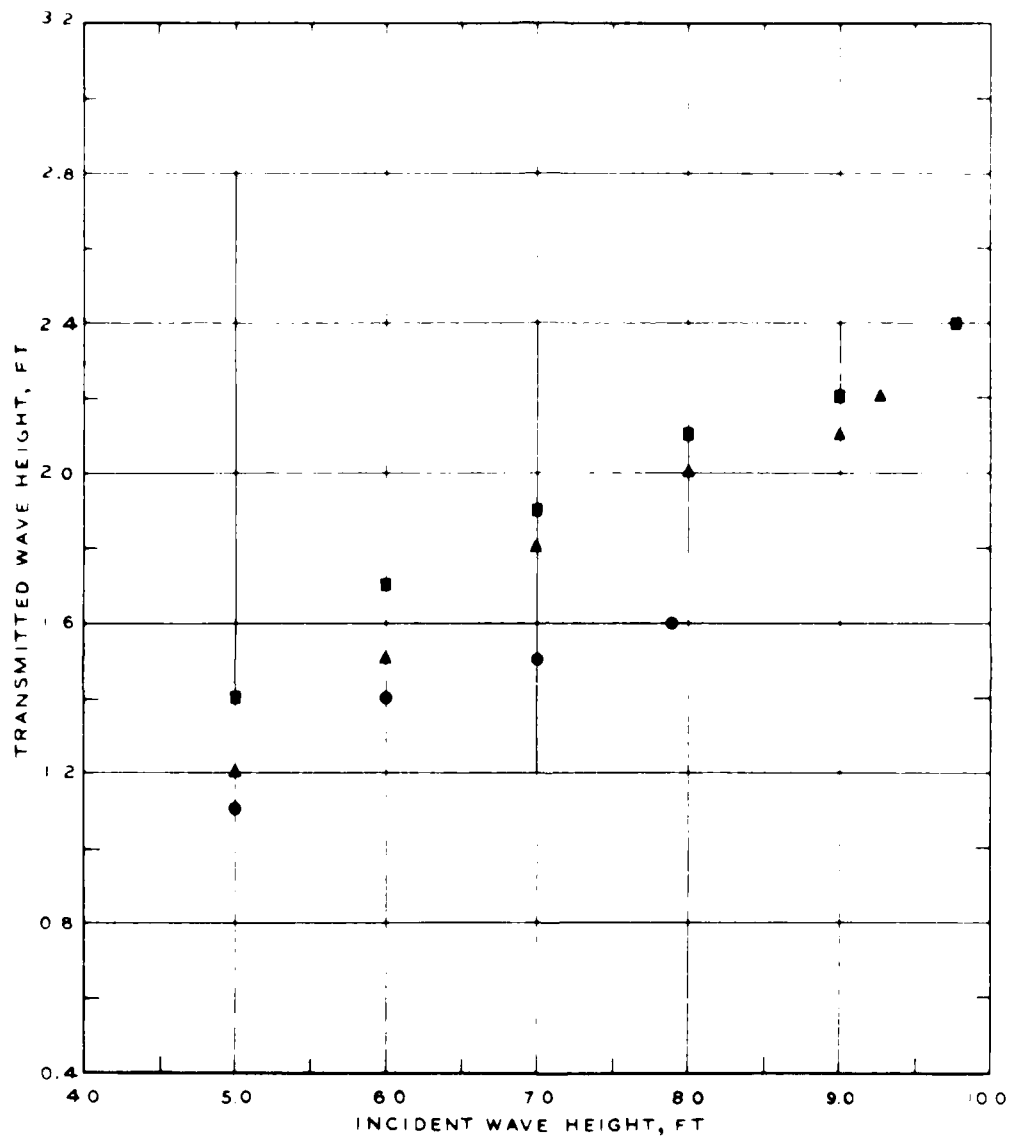
LEGEND

SYMBOL WAVE PERIOD, SEC

- 7.0
- ▲ 9.0
- 11.0

NOTE: L IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT
PLAN 1A; L



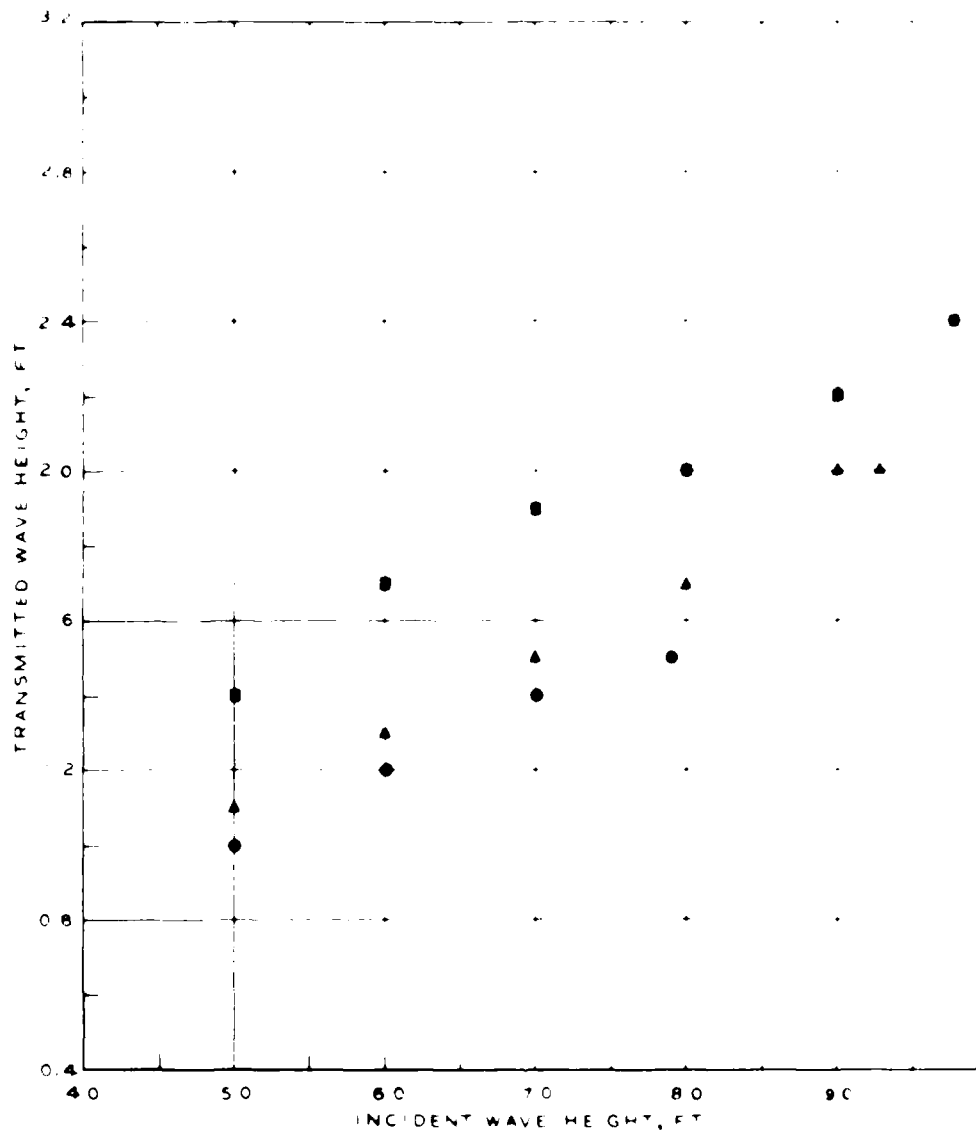
LEGEND

SYMBOL WAVE PERIOD, SEC

- 7.0
- ▲ 9.0
- 11.0

NOTE: L/2 IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED.

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT
PLAN 2; L/2



LEGEND

SYMBOL WAVE PERIOD, SEC

- 7.0
- ▲ 9.0
- 11.0

NOTE: L IS THE DISTANCE IN WAVE-LENGTHS BEHIND CENTER LINE OF THE BREAKWATER AT WHICH TRANSMITTED WAVE HEIGHTS ARE MEASURED

TRANSMITTED WAVE HEIGHT
VERSUS
INCIDENT WAVE HEIGHT

PLAN 2, L

ATE
LMED
-18